The Alamillo Bridge: *A Case Study on the Merits of Seismic Design Codes*

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Abstract

As engineers push the envelop in terms of materials, form, and construction techniques, the use of higher order analysis tools in addition to current design codes is critical. While the earthquake engineering codes may be sufficient for routine design projects, dynamic analyses, including time-history response, is necessary when designing unusual or important structures. This research uses such a structure, the renowned Alamillo Bridge in Seville, Spain, to compare and contrast the merits of existing seismic design code with more advanced dynamic analysis techniques. While a majority of new projects can be designed sufficiently using the codes, it is shown here that for this bridge, with its unique inclined-pylon design, using equivalent static analysis could lead to inefficient and/or inadequate design. Using time-history analyses from a variety of earthquakes gives great insight into the behavior of the bridge and could lead to more efficient, safe design with regard to seismic activity.

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Introduction

Current design codes—the methodologies used to approximate externally applied loading conditions in conjunction with those used to create adequate structural properties to resist these loads—are sufficient for typical civil projects in geographic areas with insignificant natural phenomena. However, when an unusual or highly important project is proposed, particularly in areas with frequent occurrence of extreme events (heavy snow load, high wind velocity, earthquakes, or other natural disasters) using a typical design approach may not be adequate. In particular, it has been shown in that past that seismic activity can be extremely detrimental to the integrity of concrete structures, even those that considered existing seismic design philosophy [12]. Therefore it is important for today's engineers to go beyond existing code and use advanced analysis methods, particularly when pushing the envelope of structural form, materials, and societal importance.

For some engineers and architects, as well as the general public, the use of concrete is merely utilitarian in nature. Its inherent strength, ease of use, and low cost lend it to a wide variety of applications, many of which are very low profile in nature. In areas where the amount of physical infrastructure is abundant, one can observe many fairly unattractive applications of concrete, including parking lots, roadways, and parking structures. It is in these projects that the existing codes; ACI, AASHTO, and ASCE to name a few, are an efficient means for structural design. The perception of concrete as solely being used in lackluster construction projects has been abandoned by Santiago Calatrava, however. Calatrava uses the properties of concrete in such a fashion as to create something that is both functional from an engineering standpoint and aesthetically pleasing. With concrete, Calatrava is able to design for structural elegance, meaning that the structure both serves the purpose of its namesake, i.e. it is structural, and also represents the form, or architecture, of the design. Perhaps the quintessential representation of using concrete for structural art is the Alamillo Bridge in Seville, Spain, with its incline pylon/cable stay design. This project proposes to evaluate the design and present an overall case study for the use of concrete in this application, and to compare traditional earthquake design methodologies with more advanced dynamic analyses. The structure is significant because it is both an engineering marvel and highly visible in the public eye. What makes it such an engineering marvel also makes it a prime candidate for a case study on the implementation of more sophisticated design and analysis techniques.

This project investigates the mechanics of the superstructure and how that relates to the choice of concrete as the load bearing material. Furthermore, it identifies characteristics at the material level. For example, research into the strength of concrete, rebar configuration, and effects of confinement are included in the study. The concepts proposed in this class- biaxial behavior, shear behavior, and ductility considerations- are used to explore this design. Finally, the structure is analyzed using more traditional Load Resistance Factored Design and then reanalyzed using time history, or dynamic analyses. Before exploring the technical details of the analysis, mechanics, and material characteristics, particularly those related to competent earthquake design, one must have a fundamental understanding of the Alamillo Bridge both in terms of how the structure behaves on a most general level and why the bridge was built to begin with.

Background

The Alamillo Bridge was built between 1987-1992 as a result of a commission for the Universal Exposition of 1992 in Seville, which coincided with the Barcelona Olympics, full membership in the European Union, and the quincentennial of the discovery of the Americas [9]. These extraordinary circumstances called for an extraordinary bridge design, out of which came this unique, inclined pylon form. The bridge has a total length of 250 m (820 ft), maximum span of 200 m (656 ft), and a mast height of 142 m (466 ft). Thirteen pairs of cable-stays support the bridge deck via a concrete-filled, steel caisson pylon inclined at 58^o [5]. These details will be illustrated in greater depth later.

The bridge consists of four basic components: the deck, cable-stays, pylon, and foundation. The live and dead loads from the deck are carried via tension upward to the pylon, which in turn transfers loads through axial compression and bending to the foundation. Most cable-stay bridges rely on front- and backstays to maintain equilibrium, and the pylon is primarily loaded in axial compression [8]. However, the Alamillo Bridge relies on the weight of the pylon, along with its incline, to resist the overturning moments produced by the unidirectional cable tension. In an ideal case, the weight of the pylon will be in perfect balance with the deck loads, which would solely result in axial loads. It should be noted that the pylon will always experience minor bending moments because its self-weight is distributed, however the cable forces act at discrete locations. Of greater importance is the fact that traffic and other external loads are constantly changing, and since the incline of the pylon cannot be adjusted, a moment will ensue. This adds fairly significant design complexity when compared to a typical cable-stay structure, since the pylon has to be designed for large axial loads and moments. Also, there are tight deflection constraints, as deflection in the pylon will directly result in deflection in the deck, which is constrained by current codes [1].

Perhaps an obvious question that arises from a discussion about the basic characteristics of the bridge is why concrete was used in the pylon design. The pylon of any cable-stayed structure has to be able to resist large compressive forces and stresses, especially near the base of the structure. Thus, a material with high compressive stress capacity is necessary. Furthermore, due to the above discussion, large bending moments can and will occur under the various loading conditions a bridge will experience. Only under one condition, the so-called “funicular loading”, will the pylon experience only axial action. Choosing the proper funicular loading scenario is not a trivial design consideration but is out of the scope of this project and will not be explored here. Regardless of this consideration, the occurrence of bending in the pylon is unavoidable and therefore a material resistant to the combined effects of axial loading and bending is necessary. A slightly more subtle design consideration is also related to funicular loading principle, and that is the distribution of weight in the pylon. Equally important to strength characteristics is the ability to maintain the balance of forces on the superstructure level. Under the geometric constraints chosen by the architect, a ratio of pylon weight to deck was calculated. Given a tower angle of 58 degrees and a cable angle of 24 degrees in the harp configuration, the weight per unit length of the pylon should be 3.4 times that of the deck [9]. A concrete pylon could provide the necessary weight along with the desired structural characteristics.

“The design requirement for a changing cross section of the pylon along its length, as well as details of the steel reinforcement, led to a composite design of steel caissons forming the outer surface of the pylon and reinforced concrete filling them” [9]. The concrete-filled caisson design serves the dual purpose of easing construction purposes as well as providing certain architectural features. Perhaps its greatest benefit is structural, however, since the steel encasement provides a constraint in-plane with the cross-section. This essentially provides a biaxial state of stress when the section is under axial loading, which has been shown to be beneficial for concrete [6].

Design Philosophy

Current design codes require that a structure have sufficient strength and/or allowable stress to resist certain loads or load combinations [2]. The ASCE specifies that a building meet three basic requirements: strength, serviceability, and self-straining forces. All loads and load combinations result from the weight of the building materials and occupants, environmental effects, and differential movement. In the case of earthquake design, an equivalent static force is applied to the building. This force is calculated via a multitude of factors, including geographic region and occupancy importance. Depending on the height of the structure, the force can be distributed through the height, relative to the amount of mass the structure is supporting at that given height.

This approach is flawed for several reasons. First, the code gives little regard to the possible motion implied by the forces a structure experiences. This has numerous ramifications, including the limited amount of acceleration that humans and certain machinery can operate under, and the fact that large displacements can cause structural materials to go beyond their elastic design limit. The second fundamental flaw of this design approach regards the lack of dynamic analysis under earthquake and other dynamic loading conditions. One could design a building with ample resistance to earthquake forces but with a fundamental period closely matching those of earthquakes that are representative of the geography in which the building resides. It has been shown in [3] and countless other publications that an excitation with a period equal to or near the fundamental frequency of the building will greatly magnify the motion of said building. The following figure shows the response of a structure at resonance, or when the frequency of the excitation equals the frequency of the structure. This kind of response could be severely detrimental to a bridge.

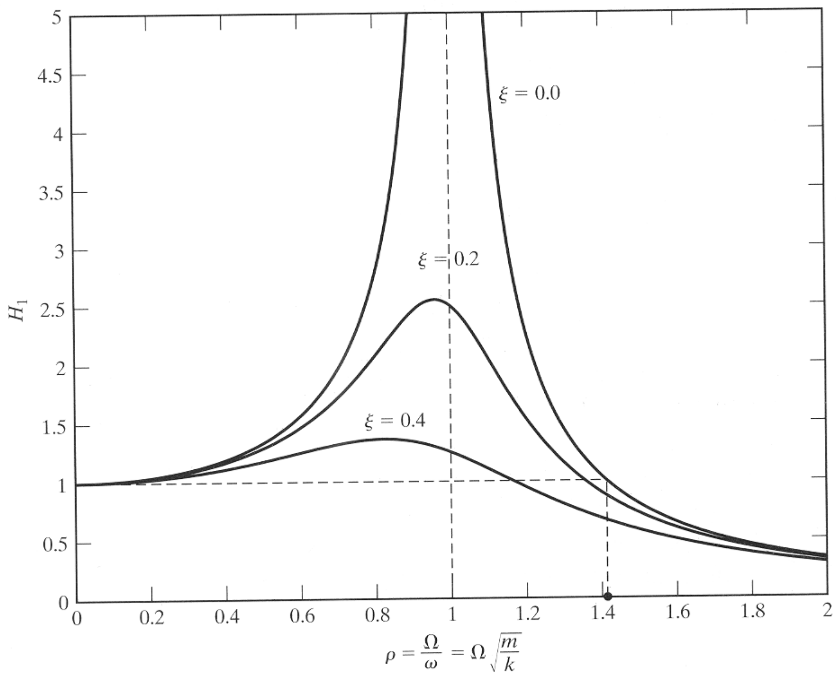

Figure 1: Illustration of Resonance [3]

The current design code "promotes a false sense of the response levels to be expected under seismic attack and typically results in severely underestimated displacements" [11]. This argument should sufficiently demonstrate the inadequacy of certain design codes for earthquake design. In this research, a different design approach is compared and contrasted, in which motion, and thus deformation, is a significant design constraint. Furthermore, the Federal Emergency Management Agency has proposed a more stringent approach to seismic design. Rather than merely designing for strength, the engineer must design to meet a performance criterion. The performance levels range from Collapse Prevention Performance to Immediate Occupancy Performance, or immediate service [4]. While the FEMA study is geared towards building design and retrofit, it is perhaps an even more viable design methodology for bridges, where the vitality of a region is tremendously affected by its infrastructure and thus a bridge should never collapse. With the current trend towards displacement-based design and the advancement in dynamic analysis procedures, it is important to explore the behavior of concrete under such conditions: large deformations and dynamic forces.

Seismic Response & Behavior of Concrete

In any civil design application, concrete cross sections are designed with certain required flexural, axial force, shear, and torsion capacity. However, for seismic design, or other dynamic loading scenarios, the ductile capacity of the concrete may be considered as the controlling design parameter for reasons described in the earlier section. Empirical relations have been developed to determine design ductility levels for typical bridge components such as cantilever columns, piers, cantilever pile shafts, and abutment piles. For short members, ductile capacities (μ) can exceed 6, while for length to depth ratios of greater than 8 a typical ductility factor is 3.5 or less. However, for more complex bridges or structures of "special importance" the relations developed in these studies may be unacceptable [11]. Thus, a more sophisticated procedure is needed after the initial design of the section. Priestley suggests, as is proposed in this project, the use of dynamic time-history analysis.

As mentioned earlier, the Alamillo bridge pylon consists of concrete-filled steel caissons, effectively acting as ideally confined concrete. Not only does confinement enable the section to resist higher compressive stress, it also allows for increases in strain capacity. This is immensely important for seismic design and coincides with the argument presented above regarding ductility: the ability of a structure to perform under large strains lends itself directly to proficient dynamic response. It has been shown that a column with confinement stress of 0.06 f_c' results in ductility ratios of greater than 4 [12], which can be considered a significant improvement over the maximum design value of less than 3.5 assumed for narrow columns.

Refer to the analysis section for a presentation of this concept, ductility capacity, with respect to the Alamillo Bridge geometry and how this relates to design using both traditional equivalent static load conditions and time-history dynamic analysis.

Analysis

The Alamillo Bridge was designed using various design codes (Spanish Code OM, British Standard BS, and Swiss code SIA) in S.I. units, combined with proprietary methods, since the structure was highly innovative. Professor Angel Aparicio of the Technical University of Barcelona used a comprehensive finite element model to analyze the bridge under applicable loading conditions and converge on an optimal design [9]. This project uses the existing geometry with certain simplifications, along with American design code, to analyze the design and perform safety evaluation. Following is a description of the finite element model, the simplifications used for analysis, and the bases for these simplifications.

SAP2000 is used for the finite element model of the superstructure. The geometry is provided by numerous sources, and [5,6,9] are referenced here. The cable attachment locations provide a convenient mesh length for both the bridge and the deck. For the pylon, one frame element is modeled between each cable connection, while in the deck two frames are used between each connection. As in any finite element model, increasing the number of elements will increase the accuracy of the analysis. However, using just one element between each element is optimal for several reasons. First, SAP2000 calculates internal forces at three discrete locations along the frame, at each end and at the center, and thus inherently provides twice the accuracy as one would assume from observing the mesh. Secondly, using fewer nodes increases the computational speed of the hardware, which is an important consideration for this model considering the power needed to run time-history analyses. The following is an illustration of the moment distribution under earthquake and dead loads, which are taken from the final construction data. Notice that the magnitude of the non-zero moments in the pylon, compared to the condition of funicular loading, which results in zero moment.

As described previously, this research provides two analytical approaches: the equivalent static design methodology and time-history dynamic analysis. To supplement the dynamic analysis and to verify behavioral trends of the bridge, three different earthquake records are used, each with slightly different duration, spectral characteristics, and magnitude. The Northridge 1994, Imperial Valley 1940, and San Fernando 1971 earthquakes are used (See Appendix B for Time-History records and various earthquake characteristics and the loads applied for equivalent static analysis). The following figures illustrate certain internal loading parameters under the Northridge 1994 earthquake, which is classified as a BSE-1, or 500-year return earthquake [4].

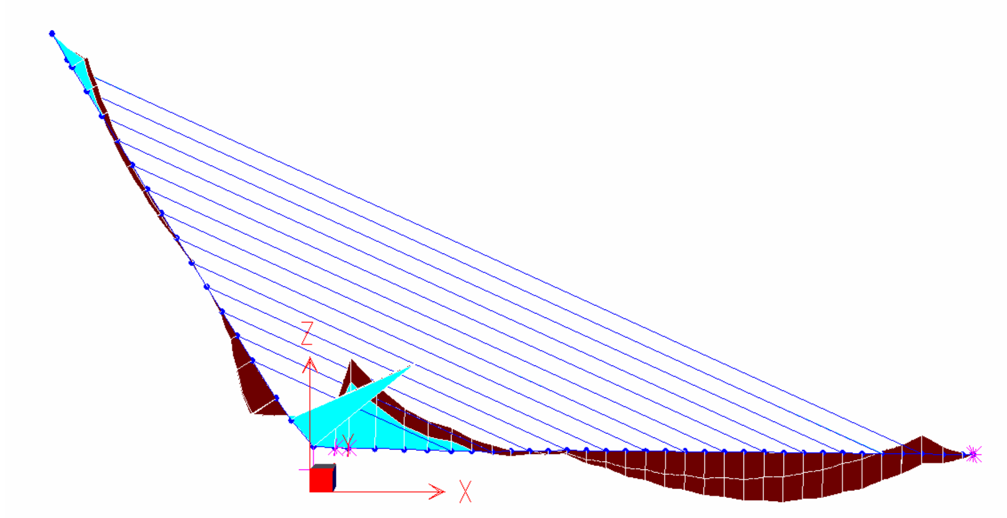

Figure 3a: Finite Element Model-Moment Distribution

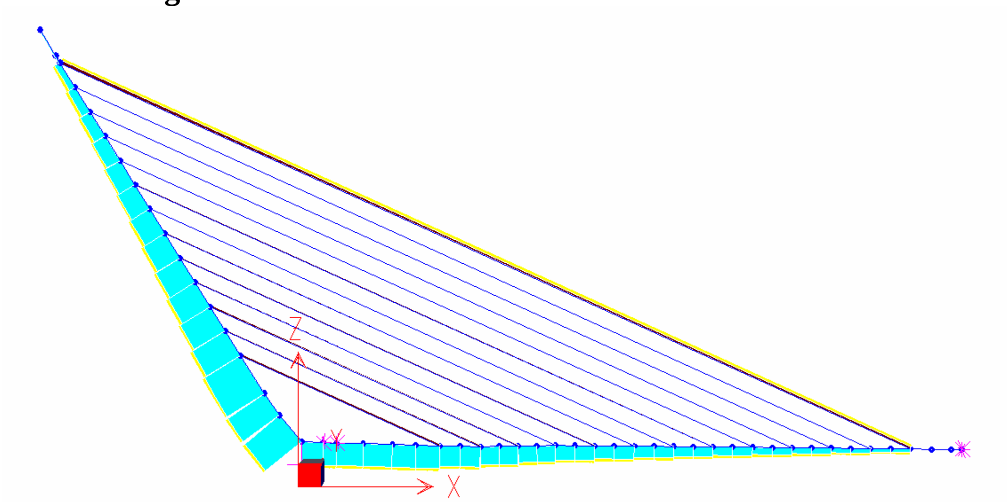

Figure 3b: Axial Force Distribution

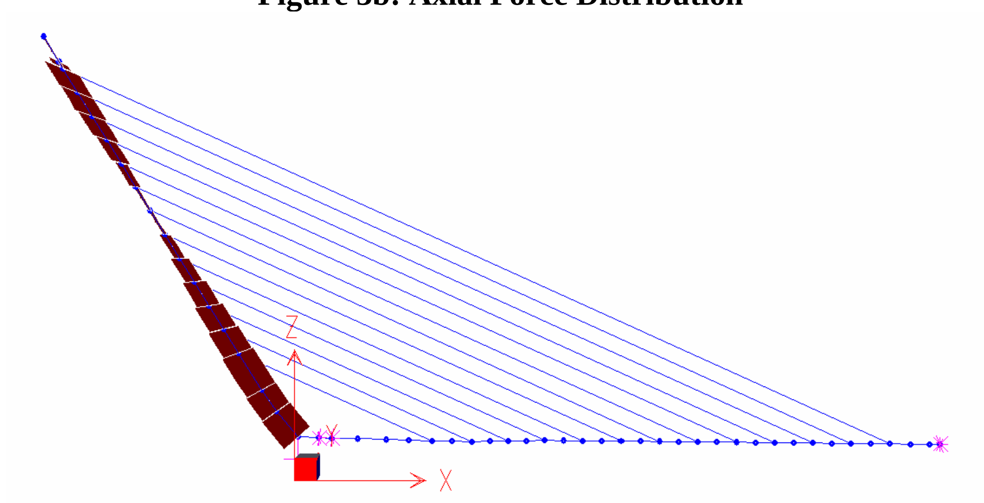

Figure 3c: Shear Force Distribution

The properties of the deck and cables, i.e. moment of inertia, modulus of torsion, area, and dead load are taken from the final design documents [9]. As a basis for analysis, an equivalent section is used for the pylon. Figure 3 shows the final design section at mid-height of the pylon. This is a relatively complex geometry, which when combined with the rebar configuration and the steel caissons, makes analysis quite difficult. Therefore, the geometric simplifications shown in Appendix A are used, combined with the concept of equivalent area for a reinforced concrete section. Reinforcement, caisson, and concrete areas are taken from final design values, and the steel area increases both the equivalent area and equivalent moment of inertia. Please refer to Appendix A for these calculations and a comparison to final design values.

Results

This section provides a quantitative comparison of the two design approaches described earlier. Results can be broken into two basic categories, internal stress (or this case allowable forces) and displacements, or deformation. Observe Figures 5a and 5b for trends regarding internal forces with respect to static and dynamic analysis procedures.

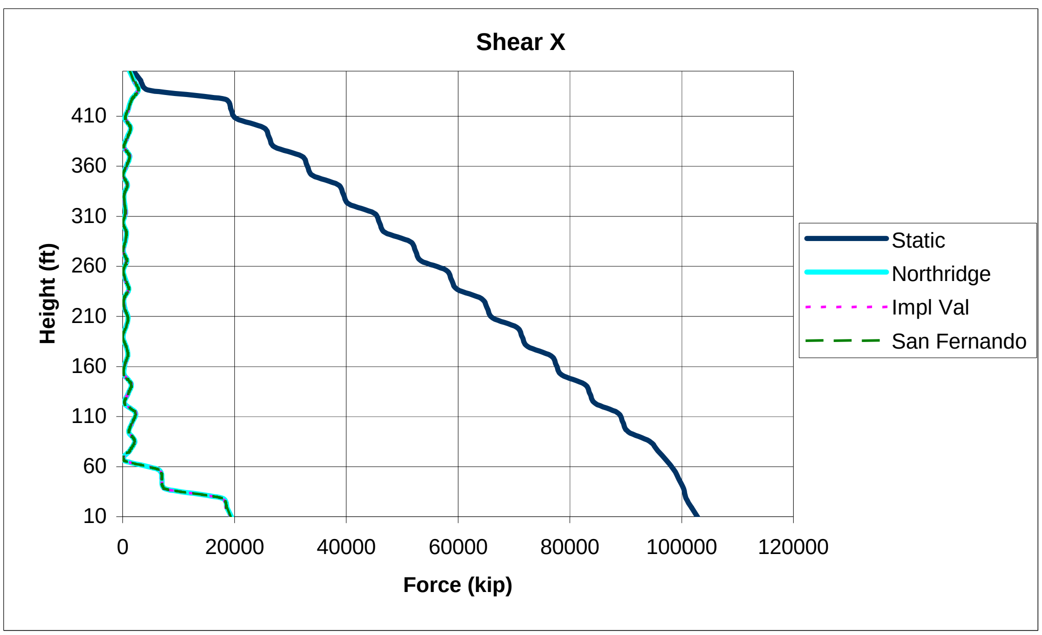

Figure 5a: Shear Capacity Demand in Pylon

It appears that the equivalent static force method is significantly over-conservative in calculating internal shear forces. By applying horizontal forces throughout the height of the structure, huge shear forces are induced in the cross-section. This can be quite accurate in building design, where large masses are concentrated at each story and forces associated with acceleration of these masses can be large. However, for bridge design, particularly for an unusual structure such as the Alamillo Bridge, this approach is highly inaccurate and could lead to inefficient design if further information is not sought.

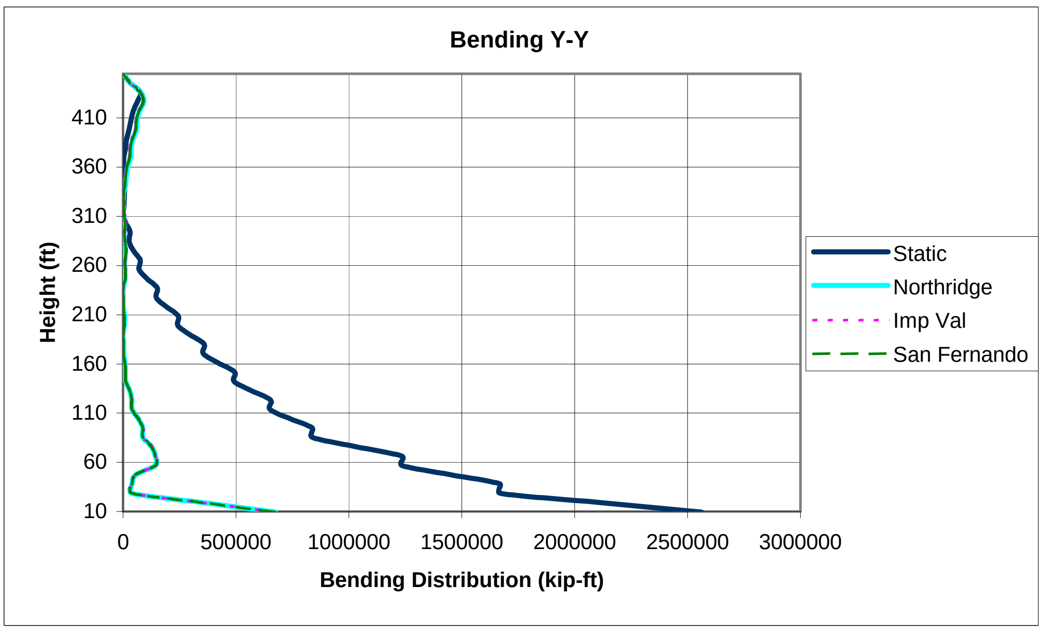

Figure 5b: Flexural Capacity Demand in Pylon

In the same way that shear capacity calculations appear to be over-conservative, so too the bending moments implied by equivalent static forces are much greater than the earthquake time-history counterpart. By applying significant forces at or near the top of the pylon, huge overturning effects result in significant bending stresses, particularly in the vicinity of the base of the structure. Given the fact that increasing shear force and flexure often necessitates increases in reinforcement, the static design approach would call for a tremendous amount of steel. This will become a significant trend in an analysis of deformation of the pylon, which will follow.

As described in the previous section called Design Philosophy, the strength-based design philosophy is no longer considered sufficient in advanced or innovative design projects, and thus an analysis of the displacements implied by seismic activity must also be considered. Though motion-based design applies to many parameters, including velocity, acceleration, and other vibration quantities, displacement is considered in this analysis because of the direct ramifications displacement has on ductility demand. Observe the displacement profile in Figure 6a, which leads directly into the drift angle, or ductility demand profile shown in Figure 6b. The drift angle profile is calculated by taking the change in displacement divided by the change in height at 17 discrete locations on the pylon.

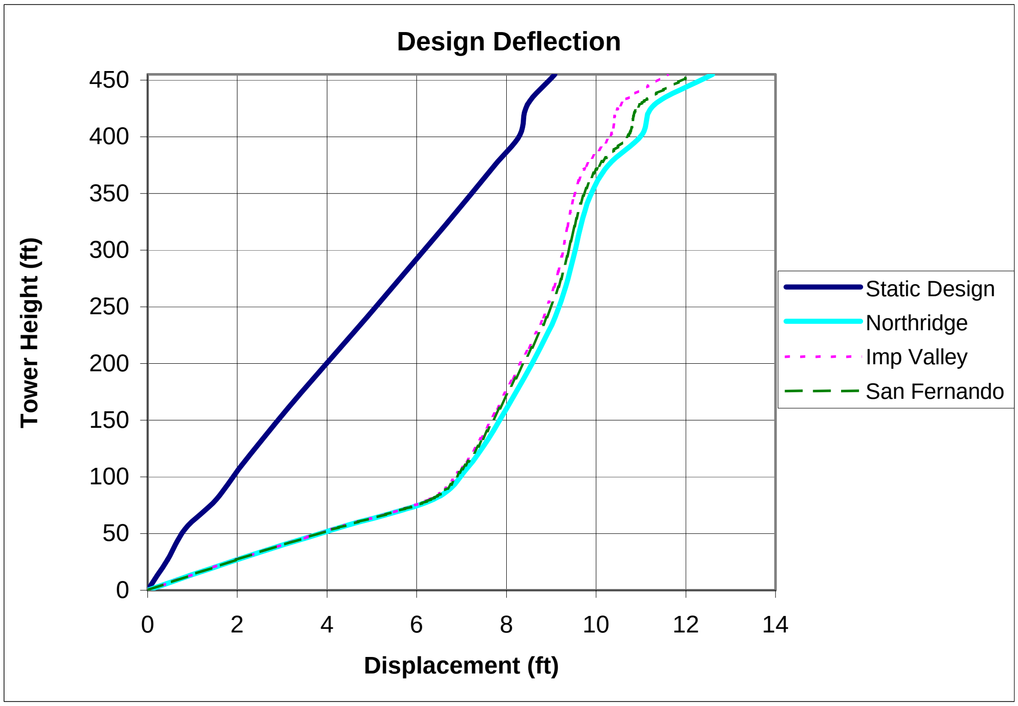

Figure 6a: Displacement Profile of Bridge Pylon

Notice first that the displacement of the pylon is significantly greater for the three earthquake time-history cases compared to the static design case. More importantly, however, is the slope of these lines, and the time-history response does not have constant slope. Greater slope denotes greater drift angle or ductility demand, which is shown in the next figure.

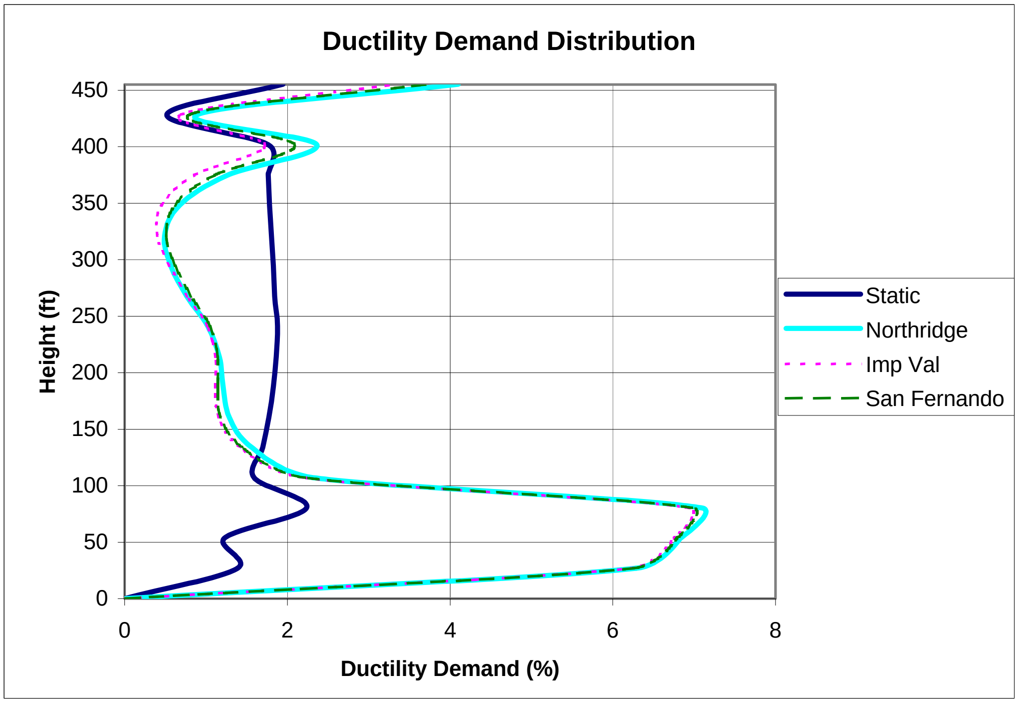

Figure 6b: Drift Angle Distribution in Pylon

This is perhaps the most significant insight into the system behavior of the pylon. The static case results in a nearly uniform ductility demand of approximately 2%, which is to be expected given the slope of the line in Figure 6a. Note the highly nonlinear behavior the structure exhibits under various earthquake records. This can be very helpful to the engineer for several reasons. First, it allows the design to be more efficient in terms of sectional properties such as moment of inertia, shape, concrete area, and amount of required reinforcement. This, combined with the results from Figures 5a and 5b, allows for optimal distribution of structural properties for seismic design. Additionally it should be stated that seismic response of this bridge is more a function of its structural properties than the content of the earthquakes themselves. For three vastly different types of earthquakes (Appendix B) the structure behaves in an extremely similar fashion both in terms of internal loading parameters and deformations.

The dichotomy of the static design process being too conservative for allowable stress design and unconservative for deflection design is a striking one. The shear and flexure cases necessitate large cross sections and perhaps high reinforcement ratios. However, increasing the amount of reinforcement has a large impact on the ductile capabilities of a section. Increasing reinforcement decreases the ductility of reinforced concrete and visa versa. To account for this phenomenon the American Concrete Institute has suggested reinforcement ratios of less than 2.5 for seismic design [13]. However, it has been shown in [11] and [12] that a more rational number for maximum reinforcement ratio in seismic design is 1.5. Anything greater could result in brittle failure under seismic events. Therefore, since implied internal forces could require large quantities of reinforcement, merely designing for strength or allowable stress is not only insufficient but could be hazardous.

Conclusions

From this analysis a fundamental question arises: why the discrepancy between the analysis procedures found in current code specifications and the dynamic analysis proposed in this research and in countless other publications? An obvious suggestion is that the static analysis is intended to be a simplified approximation of what in reality a structure might experience. The elegance of this approach lies in its simplicity and ease-of-use, and the errors associated with its implementation can be acceptable for most design projects—inaccuracies are intrinsic to this approach and are taken into account by various scale factors. Clearly in the case of the Alamillo Bridge these approximations are not sufficient, so the question needs to be explored in greater depth.

Along with the equivalent lateral force distribution described earlier, ASCE specifies that the fundamental period of a structure also meet a certain criterion. If the period is less than a certain combination of factors, one of which includes the spectral response of an equivalent seismic event, then the design is satisfactory. Please refer to Appendix C for this calculation. The Alamillo Bridge does not, in fact, meet the requirement, as the fundamental period of the bridge is 30.55 seconds and the maximum allowable period is 2.6 seconds (see Appendix C). It would seem unreasonable to increase the stiffness such that the period is decrease by more than an order of magnitude. The second and third modes should also be of great concern to the engineer. The periods of the second and third modes are 2.52 and 1.96 seconds, respectively. Though the spectral acceleration at T=30 seconds, which is not even included in Figure B-4 of Appendix B, is negligible, the spectral acceleration at T=2 seconds is quite significant, on the order of 0.5g. The Northridge Earthquake has particularly high spectral accelerations at this period, at almost 6 m/s^2 (See Figure C-5). The time-history analyses in SAP2000 illustrate the excitement of the second mode, where the dynamic response closely resembles the second mode shape shown in Appendix C, particularly for the Northridge Earthquake.

This research should illustrate the merits of a motion-based design approach and the use of time-history analyses when considering the adequacy of a design. The static, strength-based design techniques that have evolved and been used over the past several decades is still a legitimate design philosophy and should be used for a vast array of projects. However, when pushing the envelop of design, both in terms of structural elements and form, it is important to gain a better understanding of the dynamic properties of a structure. Using the existing code may not be enough and could result in either ultraconservative design or insufficient capacity, depending on the circumstances. Furthermore, it is not only important to consider the strength of the structure but also the deformations that could be induced under service loads and particularly under extreme events. In the case of concrete design, ductility can be a controlling factor.

Author’s Notes

Drawbacks to this approach and limitations to the analysis presented in this paper certainly exist. First of all, it is vital to have representative earthquake data to run accurate simulations. In many regions of the world such data does not exist, however in some cases the seismic design of a structure is more a function of the structure itself than the actual earthquake, as is the case in this study. Another limitation to the model presented here is the fact that it does not include material nonlinearity. A higher-order analysis would require the implementation of nonlinear frame analysis in SAP, which can be done and would increase the accuracy of the study. Finally, soil studies are not included in this research and should be in a real-world design scenario. Soil amplification and soil-structure interaction are two critical seismic design factors. It should also be noted that the original design of the Alamillo Bridge is indeed sufficient for safety and serviceability. Seville, Spain is not a region on the order of California in terms of seismic activity, and in fact wind is the controlling design parameter for this application.

Appendix A

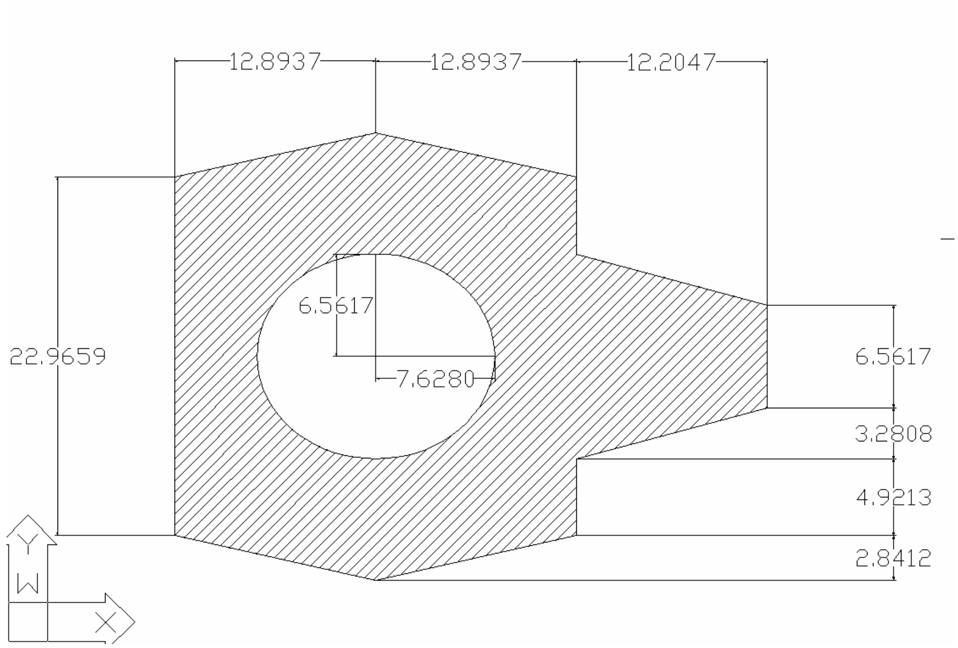

Figure A-1: Equivalent Pylon Section

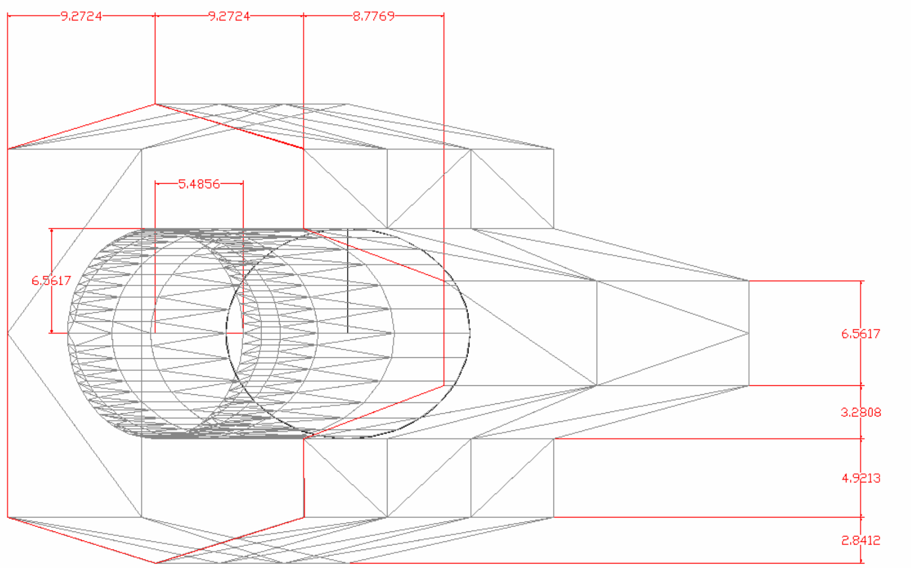

Figure A-2: Section Dimensions Normal to Neutral Axis

Cody Fleming
1.541 Project

```
> restart;
> delta:=1.04:
> A:=22.9659/2*delta:
> B:=12.8937*delta:
> C:=2.8412/12.8937*delta:
> d:=2.8412*delta:
> E:=6.5617*delta:
> F:=3.2808*delta:
> G:=12.2047*delta:
> a:=7.628*delta:
> b:=6.5617*delta:
> Ix1:=int(x^2*(d-x*C),x=0..B);
```

$$Ix1 := 522.4795212$$

```
> y1:=B/3;
```

$$y1 := 4.469816000$$

```
> A1:=d*B/2;
```

$$A1 := 19.81144030$$

```
> Ix2:=1/12*(A-b)*(B^3);
```

$$Ix2 := 1028.395630$$

```
> y2:=B/2;
```

$$y2 := 6.704724000$$

```
> A2:=B*(A-E/2);
```

$$A2 := 114.3850588$$

```
> yp:=a*sqrt(1-(x/b)^2);
```

$$yp := 7.93312\sqrt{1 - 0.02147338843\, x^2}$$

```
> y:=B-yp;
```

$$y := 13.409448 - 7.93312\sqrt{1 - 0.02147338843\, x^2}$$

```
> yc:=yp+y/2;
```

$$yc := 3.966560000\sqrt{1 - 0.02147338843\, x^2} + 6.704724000$$

```
> Ix3:=int((1/12*y^3+y*yc^2),x=0..b);
```

$$Ix3 := 4815.826810$$

```
> y3:=int(y*yc,x=0..b)/int(y,x=0..b);
```

$$y3 := 9.601689934$$

```
> A3:=int(y,x=0..b);
```

$$A3 := 48.98926983$$

```
> Ipp:=2*(Ix1+A1*y1^2+Ix2+A2*y2^2+Ix3+A3*y3^2);
```

$$Ipp := 32841.89834$$

```
> ypp:=(A1*y1+A2*y2+A3*y3)/(A1+A2+A3);
```

$$ypp := 7.237753942$$

```
> App:=A1+A2+A3;
```

$$App := 183.1857689$$

```
> Ip:=2*(Ipp+App*ypp^2);
```

$$Ip := 84876.19977$$

```
> Ix4:=int(x*(F-x*(F/G)),x=0..G);
```

$$Ix4 := 91.61840801$$

```
> y4:=G/3;
```

$$y4 := 4.230962667$$

```
> A4:=G*F/2;
```

$$A4 := 21.65427002$$

```
> Ix5:=1/12*E/2*G^3;
```

$$Ix5 := 581.4599571$$

```
> y5:=G/2;
```

$$y5 := 6.346444000$$

```
> A5:=E/2*G;
```

$$A5 := 43.30920006$$

```
> Ipt:=2*(Ix4+A4*y4^2+Ix5+A5*y5^2);
```

$$Ipt := 5610.184602$$

```
> ypt:=(A4*y4+A5*y5)/(A4+A5);
```

$$ypt := 5.641290720$$

```
> Apt:=A4+A5;
```

$$Apt := 64.96347008$$

```
> ya:=B;
```

$$ya := 13.409448$$

```
> Aa:=2*(App);
```

$$Aa := 366.3715378$$

```
> yb:=2*B+ypt;
```

$$yb := 32.46018672$$

```
> Ab:=2*(Apt);
```

$$Ab := 129.9269402$$

```
> At:=Aa+Ab;
```

$$At := 496.2984780$$

```
> yb:=(Aa*ya+Ab*yb)/(Aa+Ab);
```

$$yb := 18.39677780$$

```
> y[bar]:=2*B+G-yb;
```

$$y_{bar} := 21.11500620$$

```
> Ixx:=Ip+Aa*(ya-y[bar])^2+Ipt+Ab*(yb-y[bar])^2;
```

$$Ixx := 1.131999239\ 10^5$$

This is the final moment of inertia about the xx axis (units..feet^4).

```
> Ix6:=int(x*(E+x*F/G),x=0..G)+(A4+A5)*E^2;
```

$$Ix6 := 3758.256948$$

```
> Iyy:=Ip+2*Ix6;
```

$$Iyy := 92392.71367$$

```
> y[y]:=A+d;
```

$$y_y := 14.89711600$$

```
> conv:=3.2^4;
```

$$conv := 104.8576$$

```
> Ixx/conv;
```

$$1079.558600$$

```
> Delta1:=Ixx/conv;
```

$$\Delta 1 := 1079.558600$$

```
> PerI:=100*abs((1048.4-Delta1)/1048.4);
```

$$PerI := 2.972014500$$

```
> Delta2:=At/(3.2^2);
```

$$\Delta 2 := 48.46664824$$

```
> PerA:=100*abs((50.96-Delta2)/50.96);
```

$$PerA := 4.892762480$$

```
>
```

Table A-1: Comparison of Section Properties

	Final Design [9]	Simplified	% Error
Area (ft^2)	521.63	496.30	4.89
Inertia XX (ft^4)	113,200	109900	2.97

Appendix B

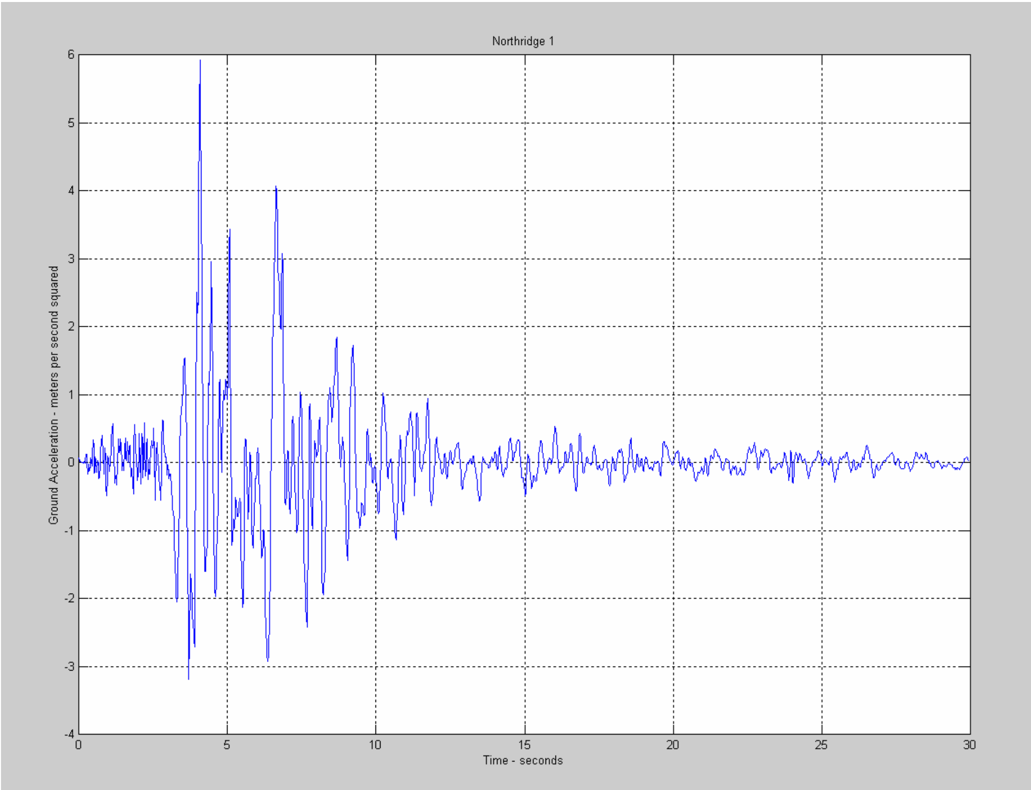

Figure B-1: Time-History of Northridge 1994 Earthquake [10]

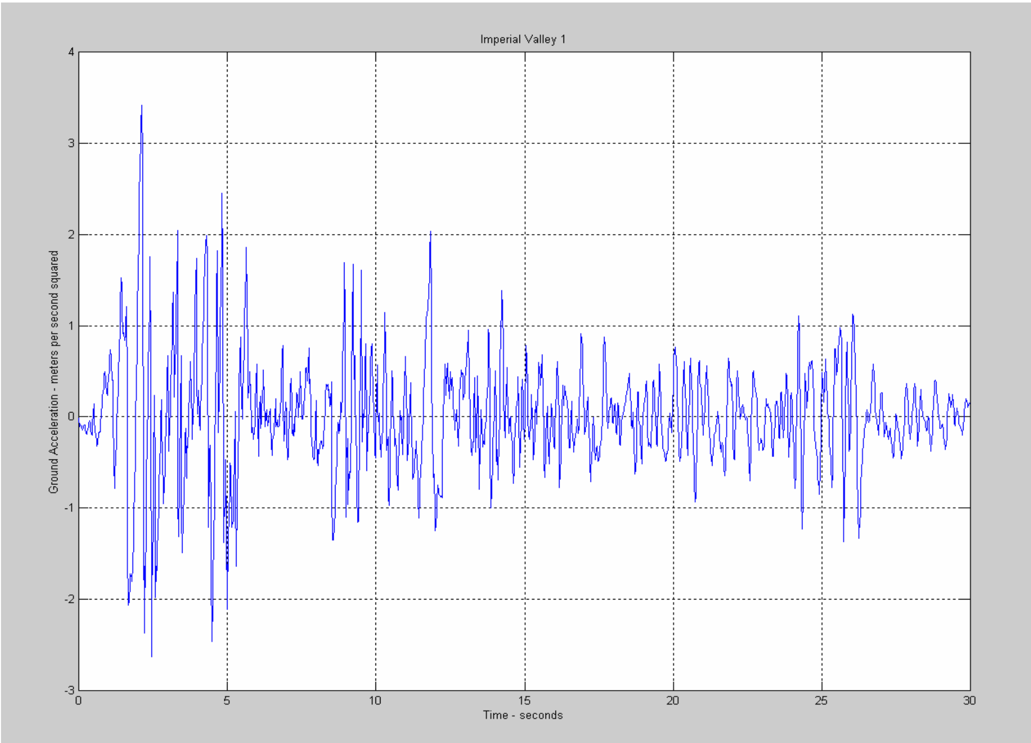

Figure B-2: Time History of Imperial Valley 1940 Earthquake

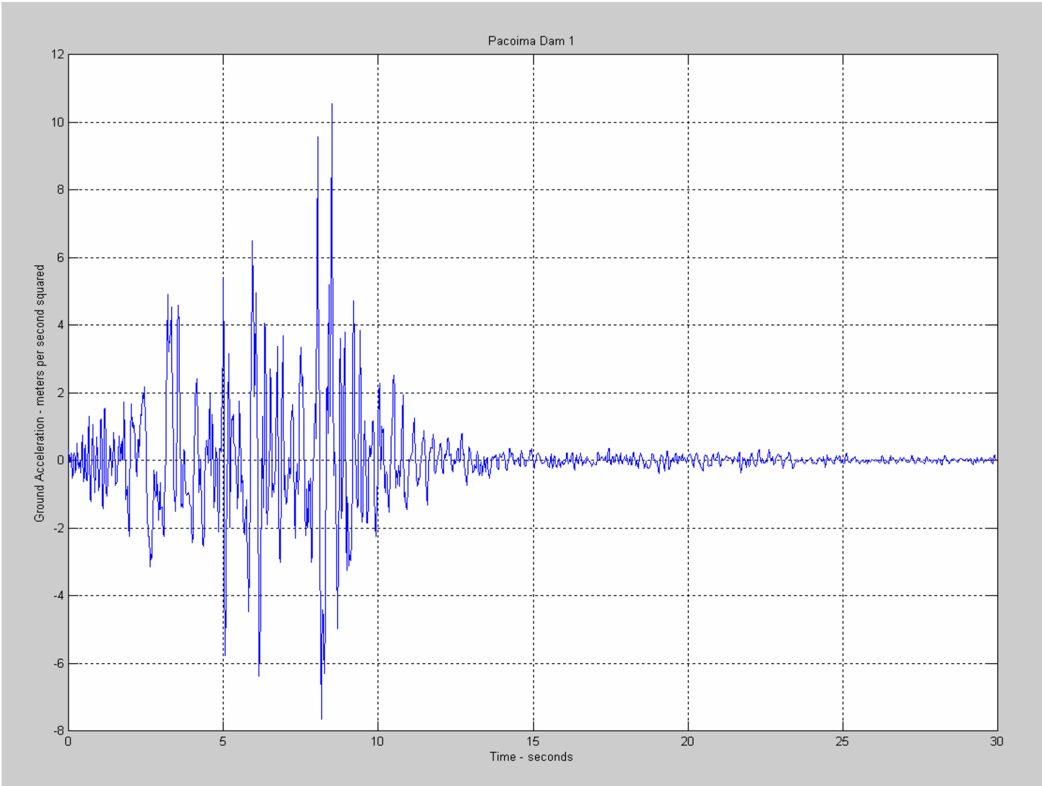

Figure B-3: Time History of San Fernando 1971 Earthquake

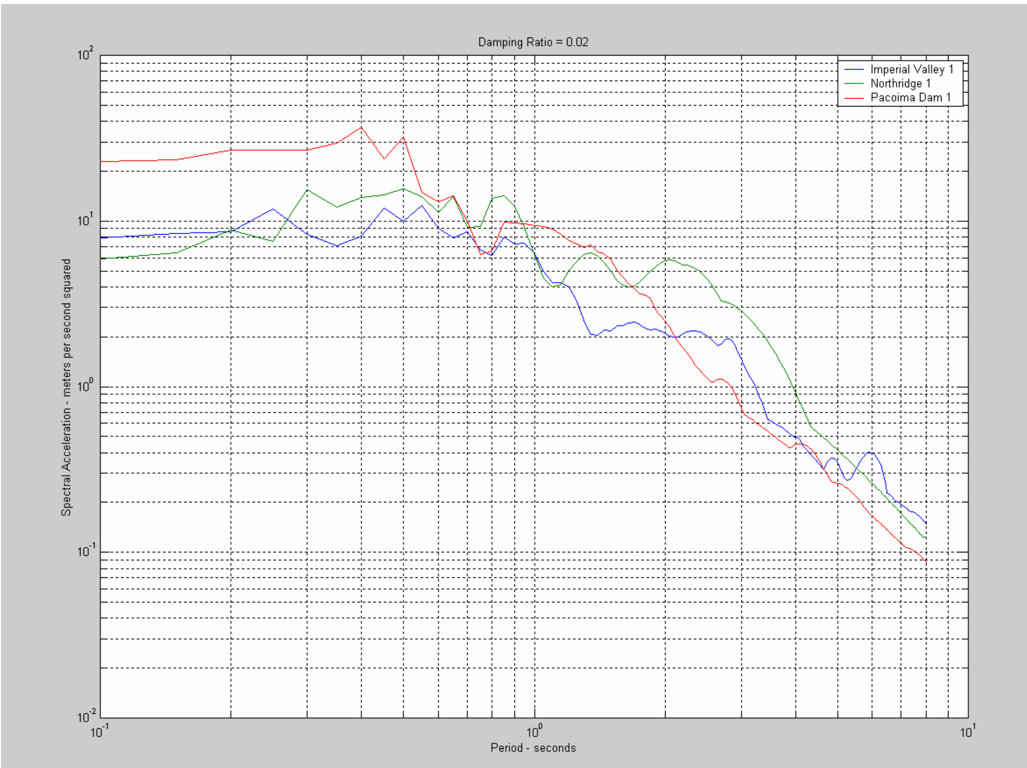

Figure B-4: Spectral Acceleration of 3 Earthquakes

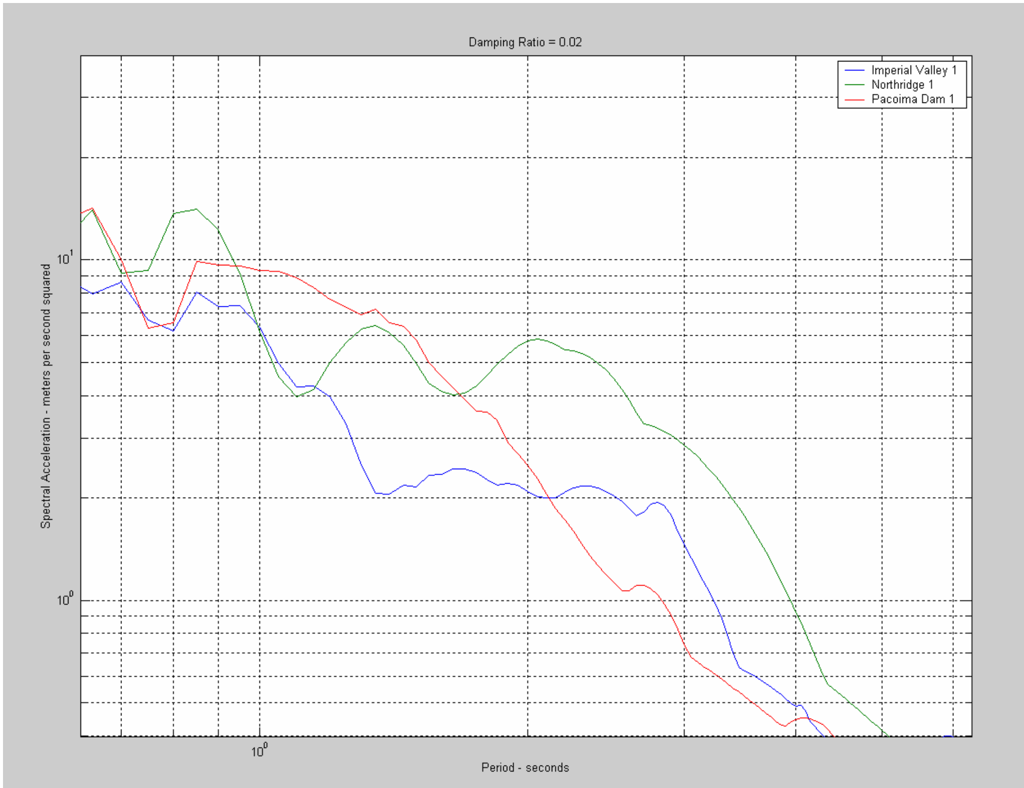

Figure B-5: Spectral Acceleration, Magnified at Period = 2 seconds

Table B-1: Static Loads [9]

Node	Height (ft)	F (kN)	F (kip)
4	26.78	537	120.7176
7	53.56	508	114.1984
10	80.34	486	109.2528
13	107.12	360	80.928
16	133.9	343	77.1064
19	160.68	327	73.5096
22	187.46	312	70.1376
25	214.24	325	73.06
28	241.02	338	75.9824
31	267.8	323	72.6104
34	294.58	308	69.2384
37	321.36	293	65.8664
40	348.14	278	62.4944
43	374.92	264	59.3472
46	401.7	165	37.092
49	428.48	180	40.464
51	455.26	144	32.3712

Appendix C

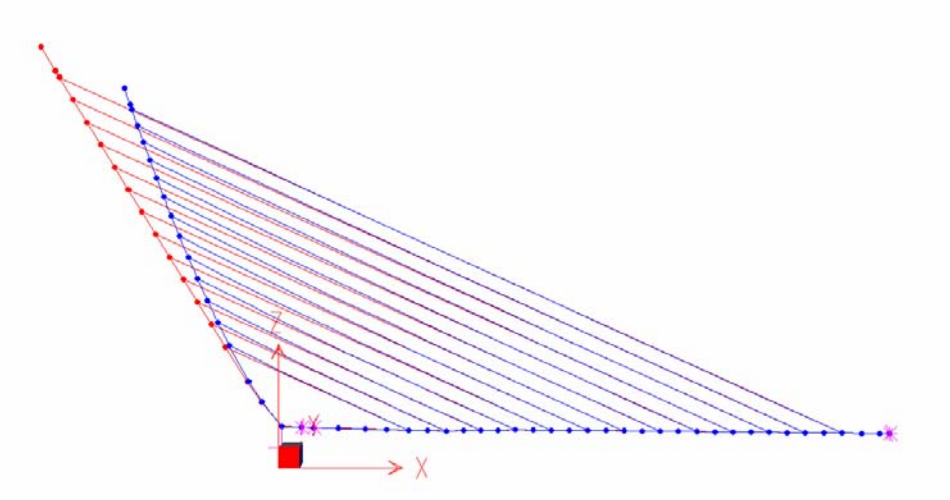

Figure C-1: 1st Mode Shape, Period = 30.55 seconds

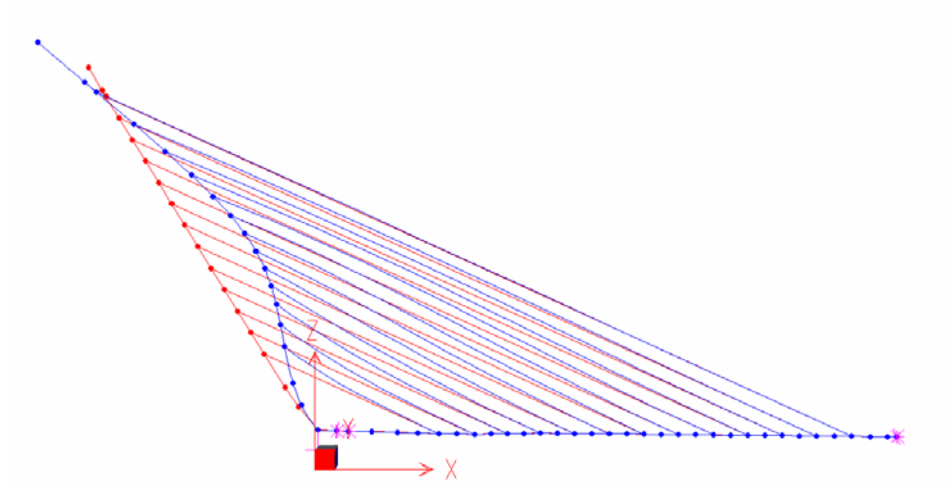

Figure C-2: 2nd Mode Shape, Period = 2.52 seconds

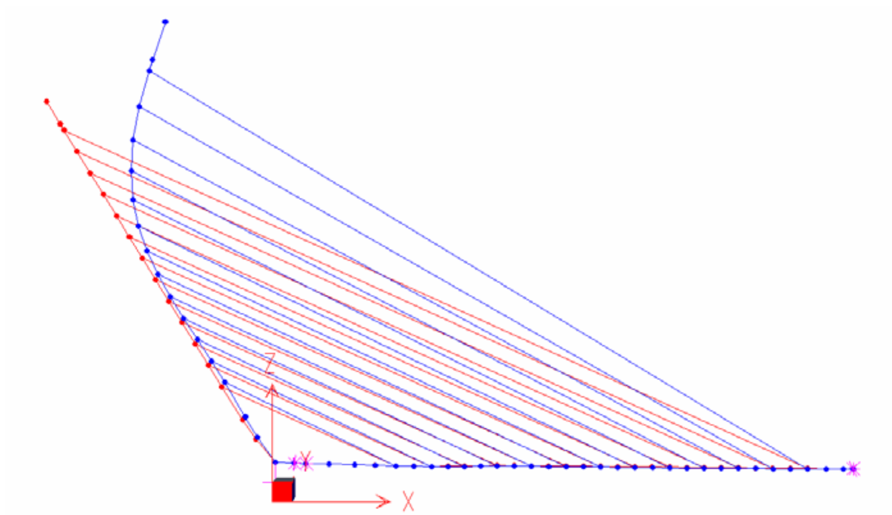

Figure C-3: 3rd Mode Shape, Period = 1.96 seconds

ASCE SPECIFICATION:

$$T_F < C_u T_a$$

$$T_a = C_T h_n^{\frac{3}{4}}$$

$C_T = 0.020$ FOR "ATYPICAL" STRUCTURES

$h_n = 455$ ft

$$T_a = 0.020 \times 455^{\frac{3}{4}}$$

$$= 1.97 \text{ sec}$$

$C_T = 1.2$, FOR $S_D \geq 0.4$

$$T_F < 1.2 \times 1.97$$

$$T_F < 2.36 \text{ sec}$$

$$T_F = 30.55 \text{ sec} \not< 2.36 \text{ sec}$$

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